Some Improvements to the Design of Sandwich Panels Subject to Local Buckling Effects

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SOME IMPROVEMENTS TO THE DESIGN OF SANDWICH PANELS SUBJECT TO LOCAL BUCKLING EFFECTS

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ABSTRACT

Past research into the local buckling behaviour of fully profiled sandwich panels has been based on polyurethane foams and lower grade steels, and not for very slender plates. The Australian sandwich panels use polystyrene foam and thinner (0.42 mm) and high strength steels (G550 with a minimum yield stress of 550 MPa), which are bonded together using separate adhesives. Therefore a research project on Australian sandwich panels was undertaken using experimental and finite element analyses. The experimental study on 50 foam-supported plate elements and associated finite element analyses produced a large database for sandwich panels subject to local buckling effects, but revealed the inadequacy of conventional effective width formulae for panels with slender plates. It confirmed that these design rules could not be extended to the slender plates in their present form. In this research, experimental and analytical results were used to improve the design rules. This paper presents the details of experimental and finite element analyses, their results and the improved design rules.

INTRODUCTION

The use of sandwich panels in the construction of building structures offers many advantages because of the light, cost effective and durable structures they can generate. Until recently sandwich panel construction in Australia has been limited to cold-storage buildings due to the lack of design methods and data. However, in recent times, the sandwich panels are increasingly used in building structures particularly as roof and wall cladding systems.

Structural sandwich panels consisting of two strong facings separated by and bonded rigidly to the centre core of lighter and weaker material provide a composite construction with high bending stiffness and minimum weight. Sandwich panels generally used in Australia comprise of thinner (0.42 mm) and high strength (minimum yield stress of 550 MPa and reduced ductility) steel faces and relatively thick polystyrene foam core which are bonded together using separate adhesives. The steel faces of sandwich panels are generally used in three forms: flat, lightly profiled, and profiled. The faces of sandwich panels provide architectural appearance, structural stiffness, and protect the relatively vulnerable core material against damage or weathering. The faces take compressive and tensile loads and the core transfers shear loads between the faces while providing high bending stiffness. Hence, sandwich panels represent an excellent example of the optimum use of dissimilar materials.

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The fully profiled sandwich panels are subjected to local buckling effects (Figure 1) under wind pressure loading. The plate elements of sandwich panels are supported by polystyrene foam and therefore their local buckling behaviour is significantly better than flat plate elements. Buckling of the panels may occur at a stress level lower than the yield stress of steel, but the panels will have considerable postbuckling strength. Such local buckling and postbuckling problems are treated for design purposes by utilising the effective width principles.

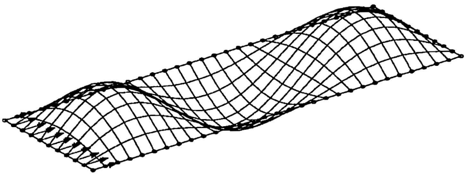


Figure 1: Local Buckling of Sandwich Panels

Past research in Europe and the USA (Davies 1987, 1990, 1992, 1993, Hassinen 1995, ECCS, 2000) has investigated the local buckling behaviour and developed modified conventional effective width rules for the plate elements in sandwich panels. However, these studies have been based on polyurethane foams and lower grade steels, and rely on some empirical factors. Moreover, these rules can be applied only for low width to thickness (*b/t*) ratios (Figure 2) of the plate elements. But in the sandwich panel construction, *b/t* ratios can be as large as 600 (Mahendran and Jeevaharan, 1999). Therefore a research project was conducted using a series of experiments and numerical analyses to study the local buckling behaviour of sandwich panels made of high strength steel faces and polystyrene foam covering a wide range of *b/t* ratios.

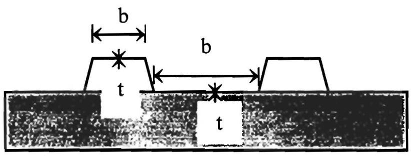


Figure 2: Critical *b/t* Ratios of Profiled Sandwich Panels for Local Buckling

In the first phase, a detailed experimental study on 50 foam-supported plate elements was conducted. The results revealed the inadequacy of conventional effective width formulae. To eliminate this problem and to improve the understanding of local buckling behaviour further, finite element analyses (FEA) of sandwich panels using ABAQUS were undertaken. Both FEA and experimental results were then used to review the current design rules. This paper presents the details of the FEA and the results compared with relevant experimental results.

LOCAL BUCKLING THEORY

Thin steel faces supported by a thick foam core can be considered as a plate on elastic foundation as shown in Figure 3. A simply supported rectangular plate is subject to an applied stress p along the two transverse edges. The longitudinal edges of the plate are assumed to be simply supported. The length of the plate in x-direction is large compared with the width. The critical buckling stress σ_{cr} of this plate is given by (Davies and Hakmi, 1990):

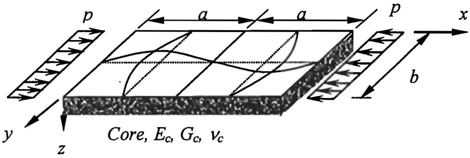


Figure 3: Steel Plate in Compression with Core as an Elastic Foundation

$$\sigma_{cr} = K\frac{\pi^2 E_f}{12(1-\nu_f^{\,2})}\left[\frac{t}{b}\right]^2 \tag{1}$$

where K is the buckling coefficient and given by

$$K = \left[\frac{1}{\phi}+n^2\phi\right]^2 + R\phi\left[1+n^2\phi^2\right]^{\frac{1}{2}} \tag{2}$$

and $$R = \frac{24(1-\nu_f^{\,2})(1-\nu_c)E_c}{\pi^3(1+\nu_c)(3-4\nu_c)E_f}\left[\frac{b}{t}\right]^3 \quad \text{(actual)} \tag{3}$$

$$R = \frac{12(1-\nu_f^{\,2})}{\pi^3}\frac{\sqrt{E_c G_c}}{E_f}\left[\frac{b}{t}\right]^3 \quad \text{(simplified)} \tag{4}$$

where E_f and E_c are Young's modulus of steel faces and core, respectively, G_c is the shear modulus of foam core, ν_f and ν_c are the Poisson's ratio for steel face and foam core, respectively, b is the width and t is the plate thickness. The composite action between faces and core is modelled by the dimensionless stiffness parameter R. The critical buckling stress σ_{cr} can be found by minimising the buckling coefficient K with respect to the wavelength parameter ϕ. Hence the condition $\partial K/\partial\phi = 0$ from Equation 2 gives:

$$2n^4\phi - \frac{2}{\phi^3} + R(2n^2\phi^2+1)(n^2\phi^2+1)^{-\frac{1}{2}} = 0 \tag{5}$$

The value of ϕ, which is a ratio of half-wave buckle length a to the width of plate element b, can be determined from Equation 5 using a suitable numerical method. Using ϕ into Equation 2, K can be evaluated. As this process is complicated and hence a number of explicit mathematical formulae have been proposed to determine K for sandwich panels with profiled faces. They are given next.

1. By Hassinen (1991)

$$K = 4 - 0.415R + 0.703R^2 \text{ with } R = \frac{b}{t}\left[\frac{E_c}{E_f}\right]^{1/3} \tag{6}$$

2. By Hassinen (1991)

$$K = 4 - 0.474R + 0.985R^2 \quad \text{with}\, R = \frac{b}{t}\left[\frac{E_c G_c}{E_f^{\,2}}\right]^{1/6} \tag{7}$$

3. By Davies and Hakmi (1990)

$$K = [16 + 11.8R + 0.055R^2]^{1/2} \quad \text{with} \quad R = \frac{12(1-\nu_f^{\,2})\sqrt{E_c G_c}}{\pi^3 E_f}\left[\frac{b}{t}\right]^3 \tag{8}$$

4. By Davies and Hakmi (1990) for design purposes by replacing R with $0.6R$

$$K = [16 + 7R + 0.02R^2]^{1/2} \tag{9}$$

5. By Mahendran and Jeevaharan (1999) to include R from 0 to 600

$$K = [16 + 4.76R^{1.29}]^{1/2} \qquad \text{with} \quad R = \frac{12(1-\nu_f^{\,2})\sqrt{E_c G_c}}{\pi^3 E_f}\left[\frac{b}{t}\right]^3 \tag{10}$$

In the current European Recommendations for Sandwich Panels, Part I: Design (ECCS, 2000), the following formulae have been recommended for predicting the value of K. These expressions are applicable for $0 \le R \le 200$ and $b/t \le 250$, and are based on an empirical reduction factor of $0.6R$.

$$K = [16 + 7R + 0.02R^2]^{1/2} \text{with} \quad R = 0.35\frac{\sqrt{E_c G_c}}{E_f}\left[\frac{b}{t}\right]^3 \tag{11}$$

DESIGN RULES

The critical buckling stress itself does not provide any satisfactory basis for design, but it can be used as a useful design parameter. It is well known that in cold-formed steel design, the b/t ratios are usually large, hence local buckling becomes a major design criterion for the compression members. Buckling of these elements may occur at a stress level lower than the yield stress, thus, post-buckling behaviour is important. In the design of cold-formed steel members, such local buckling problems are treated by utilising the effective width principles. In this method, the width b of the compressed plate element is replaced by a reduced effective width, b_{eff}. The design formula takes the form:

$$\left.\begin{aligned} & b_{eff} = \rho b \\ & \rho = \frac{1}{\lambda}\left[1 - \frac{0.22}{\lambda}\right] \text{ for } \lambda > 0.673 \\ & \rho = 1.0 \text{ for } \lambda \le 0.673 \\ & \lambda = 1.052\left[\frac{b}{t}\right]\sqrt{\frac{f_y}{E_f K}} \end{aligned}\right\} \tag{12}$$

where f_y = yield stress, E_f = Young's modulus, t = plate thickness, K = buckling coefficient. This effective width approach can be extended to the profiled faces of sandwich panels by using the modified values of the buckling coefficient K evaluated from Equations 2 or 6 to 11. The effective width evaluated in this way is applicable only for the plate elements with low b/t ratios. For slender plates with high b/t ratios, this design formula overestimates the effective widths thus making it unsafe for design purposes. Hence there is a need to improve the effective width approach for the sandwich panels with slender plates.

EXPERIMENTAL INVESTIGATION

In the experimental investigation, 50 steel plate elements (25 for G550 grade steel and 25 for G250 grade steel) supported by polystyrene foam cores as used in the profiled sandwich panels were tested under compression load. As the foam thickness has negligible effect on the buckling strengths (Mahendran & Jeevaharan 1999, Mahendran & McAndrew 2000), a constant thickness of 100 mm was used throughout the tests. To cover a large range of b/t ratios (between 50 to 500), both the thickness and width of the plates were varied for each grade of steel (see Table 1). The lengths of the plates were chosen as three times the width (b) plus 10 mm for clamping. The steel faces and foam were glued to each other by using a suitable adhesive. The specimens were tested at least after 48 hours of attachment to ensure the adhesive was set and steel face and foam were joined properly. Details of the experimental program and test specimens are given in Table 1.

A specially constructed test rig was used to hold the test specimen with two vertical clamps allowing the vertical displacement and free rotation at the longitudinal edges, as required for the simply supported conditions. The test specimens were placed in the test rig between two loading blocks. A schematic diagram of the test rig is given in Figure 4.The compression tests of the steel plate elements were carried out using a Tinius Olsen Testing Machine. A compression load was applied at a constant rate of 0.5 mm/min until the specimen failure. The buckling and ultimate loads of each test specimen were recorded. The buckling load was noted by visual observation of plate buckling whereas the maximum load carried by the specimen was taken as the ultimate load. Hence the buckling load was approximate, but the ultimate load could be considered exact.

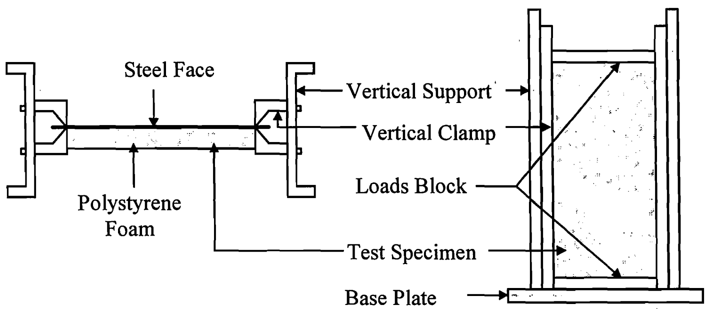


Figure 4: Schematic Diagram of Test Rig

Table 1: Experimental Program

Test series	Plate width b (mm)	G550 steel plates					G250 steel plates				
		Thickness (mm)		Measured		b/t ratio	Thickness (mm)		Measured		b/t ratio
		Spec.	bmt	f_y (MPa)	E_f (GPa)		Spec.	bmt	f_y (MPa)	E_f (GPa)	
1	50	0.95	0.95	637	226	52.6	1.00	0.93	326	216	53.8
2	50	0.80	0.80	656	230	62.5	0.80	0.73	345	217	68.5
3	50	0.60	0.60	682	235	83.3	0.60	0.54	360	218	92.6
4	50	0.42	0.42	726	239	119.0	0.40	0.39	368	220	128.2
5	80	0.95	0.95	637	226	84.2	1.00	0.93	326	216	86.0
6	80	0.80	0.80	656	230	100.0	0.80	0.73	345	217	109.6
7	80	0.60	0.60	682	235	133.3	0.60	0.54	360	218	148.1
8	80	0.42	0.42	726	239	190.5	0.40	0.39	368	220	205.1
9	100	0.95	0.95	637	226	105.3	1.00	0.93	326	216	107.5
10	100	0.80	0.80	656	230	125.0	0.80	0.73	345	217	137.0
11	100	0.60	0.60	682	235	166.7	0.60	0.54	360	218	185.2
12	100	0.42	0.42	726	239	238.1	0.40	0.39	368	220	256.4
13	120	0.95	0.95	637	226	126.3	1.00	0.93	326	216	129.0
14	120	0.80	0.80	656	230	150.0	0.80	0.73	345	217	164.4
15	120	0.60	0.60	682	235	200.0	0.60	0.54	360	218	222.2
16	150	0.95	0.95	637	226	157.9	1.00	0.93	326	216	161.3
17	150	0.80	0.80	656	230	187.5	0.80	0.73	345	217	205.5
18	150	0.60	0.60	682	235	250.0	0.60	0.54	360	218	277.8
19	150	0.42	0.42	726	239	357.1	0.40	0.39	368	220	384.6
20	180	0.60	0.60	682	235	300.0	0.60	0.54	360	218	333.3
21	180	0.42	0.42	726	239	428.6	0.40	0.39	368	220	461.5
22	200	0.95	0.95	637	226	210.5	1.00	0.93	326	216	215.1
23	200	0.80	0.80	656	230	250.0	0.80	0.73	345	217	274.0
24	200	0.60	0.60	682	235	333.3	0.60	0.54	360	218	370.4
25	200	0.42	0.42	726	239	476.2	0.40	0.39	368	220	512.8
Note: f_y – measured yield stress of steel, E_f – measured Young's modulus											
b/t ratio – plate width b/bmt, Spec. – specified thickness											
bm$t$ – estimated base metal thickness based on measured total coated thickness											

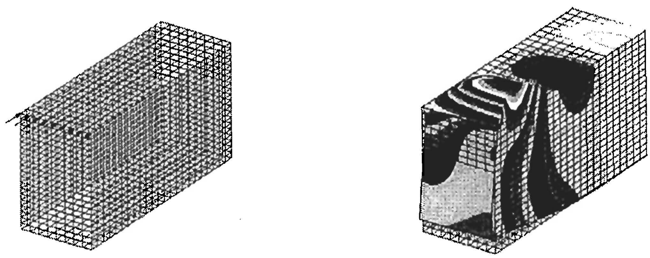
FINITE ELEMENT ANALYSIS

Local buckling behaviour of sandwich panels was also investigated using a finite element program ABAQUS. The finite element model was based on the application of compressive load to one end of the steel face with all four sides of the plate being simply supported. The foam core was extended sufficiently deep to ensure that the theoretical approach of elastic half space (i.e. the core) to extend infinitely in this direction was simulated. To achieve this, a constant depth of 100 mm, same as in the experiments, was used for all the models.

The steel plate was modelled using S4R5 shell elements with four nodes and five degree of freedom per node whereas the foam core was modelled using C3D8 3D solid elements with eight nodes and three degree of freedom per node. Since there was no relative movement between the steel faces and foam core, they were modelled as a single unit. In order to determine the appropriate mesh density, a convergence study was conducted with gradually increasing mesh size. On the basis of convergence study, a mesh with 10 mm square surface elements (for steel plate) and solid elements with 10×10×5 mm throughout the foam depth was used. Measured material properties of polystyrene foam and steel faces were used in the analysis. They are E_c = 3.8 MPa, G_c = 1.76 MPa, ν_c = 0.08 for foam whereas the values for both G550 and G250 grades of steels are given in Table 1. The Poisson's ratio of steel was assumed to be $\nu = 0.3$. Both materials were considered to be isotropic. A series of elastic buckling and non-linear analyses was undertaken using two different types of finite element models. The first model was the half-length experimental model analysed to calibrate with the experimental results whereas the second model was the half-wave buckle model analysed to simulate the real conditions of the sandwich panels used in building structures.

Half Length Experimental Model

To simulate the sandwich panels tested in the laboratory, a half-length model was used with appropriate boundary conditions including that of symmetry. To confirm the results, a full length model was also analysed for some of the specimens. The length of the model used was 3 times the width as used in the experiments. Since the results from full length and half-length models agreed well, further analyses were conducted using half-length models considering only half width to save on computational time.



(a) Model (b) Buckle shape

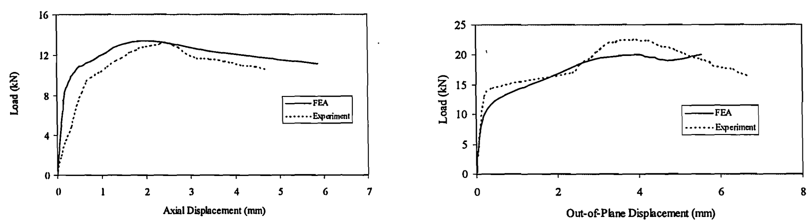
Figure 5: Half-Length Experimental FEA Model of Steel Plate with Foam Core

Figure 5a shows the model geometry, mesh size and the loading pattern for half-length model. Appropriate boundary conditions were applied only to the steel face at the loading end and one of the longitudinal edges to simulate the experiments whereas symmetric boundary conditions were applied to the entire surface (i.e., to the steel faces and foam core) along both the longitudinal direction and across the width. The model was first analysed using an elastic buckling analysis. The first buckling mode which was very close to the experimental buckling mode was used to input geometric imperfection for the non-linear analysis. For all non-linear analyses, 0.1 times the plate thickness ($0.1t$) was used as the magnitude of geometric imperfection. Figure 5b shows the buckled shape of the half-length model.

Buckling and ultimate loads were obtained from elastic buckling analysis and non-linear analysis, respectively. These FEA results were compared with the corresponding experimental results. As seen in Table 2, the results from FEA and experiments agreed reasonably well for both G550 and G250 steel plates. The mean values of the ratio of FEA and experimental buckling and ultimate stresses were found to be 1.00 and 0.94, respectively, for G550 steel plates and 1.05 and 0.93, respectively, for G250 steel plates. The corresponding coefficients of variation (COV) were 0.06 and 0.11, respectively, for G550 steel plates and 0.08 and 0.12, respectively, for G250 steel plates. Figure 6 presents the comparison of typical load-deflection curve from FEA and experimental results. Figure 7 presents the comparison of ultimate strength results from FEA with experiments. All these comparisons confirm that half-length models can be successfully used to represent the local buckling behaviour of experimental sandwich panels.

Table 2: Comparison of FEA Results based on Half-Length Model with Experimental Results

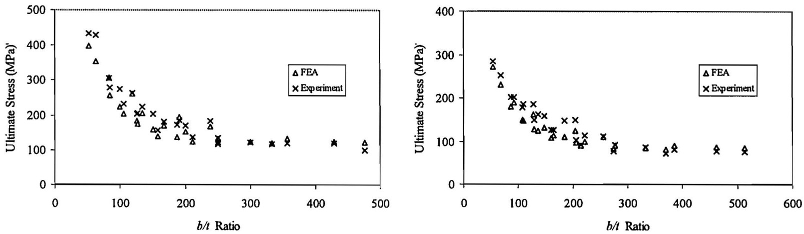
Test No.	G550 Steel Plates					G250 Steel Plates				
	b/t ratio	Buckling stress (MPa)		Ultimate stress (MPa)		b/t ratio	Buckling stress (MPa)		Ultimate stress (MPa)	
		FEA	Expt.	FEA	Expt.		FEA	Expt.	FEA	Expt.
1	52.6	352.8	336.2	397.5	434.3	53.8	327.7	272.5	271.2	285.4
2	62.5	275.0	293.0	353.0	428.3	68.5	232.1	221.9	231.2	251.5
3	83.3	196.0	238.3	308.3	305.0	92.6	167.4	151.9	188.5	201.5
4	119.0	138.6	141.9	260.0	264.3	128.2	125.1	146.7	162.6	186.2
5	84.2	182.2	170.3	257.1	279.2	86.0	172.6	171.2	180.9	201.6
6	100.0	153.1	155.8	223.0	275.0	109.6	134.6	152.6	148.1	185.4
7	133.3	121.3	112.9	205.0	223.3	148.1	110.0	98.1	132.2	159.0
8	190.5	103.3	97.0	194.0	186.0	205.1	97.4	88.1	124.0	149.0
9	105.3	140.6	139.4	203.3	232.6	107.5	133.8	123.4	149.6	178.1
10	125.0	122.5	132.8	182.9	203.4	137.0	111.8	121.4	123.6	162.1
11	166.7	105.0	102.0	171.3	181.8	185.2	97.0	90.9	110.4	148.1
12	238.1	93.6	87.6	167.6	184.0	256.4	89.2	76.9	109.2	111.3
13	126.3	118.4	122.1	174.6	205.2	129.0	113.7	119.4	128.5	150.2
14	150.0	107.4	119.8	159.1	203.5	164.4	99.5	98.6	113.0	126.4
15	200.0	95.1	91.0	152.2	169.9	222.2	89.0	85.2	99.4	113.3
16	157.9	101.1	104.6	140.4	158.1	161.3	97.7	96.8	108.7	125.5
17	187.5	93.9	96.1	138.3	172.8	205.5	88.3	87.2	96.5	101.9
18	250.0	86.1	83.9	133.2	133.9	277.8	81.5	76.5	89.4	91.2
19	357.1	81.1	79.0	133.3	119.2	384.6	78.1	67.2	90.6	80.3
20	300.0	80.5	78.6	124.4	122.6	333.3	76.7	73.6	84.7	86.0
21	428.6	77.1	77.8	124.9	118.4	461.5	74.6	64.1	86.6	78.3
22	210.5	86.2	88.2	124.3	136.5	215.1	83.8	84.9	89.8	91.6
23	250.0	82.1	79.9	122.0	117.1	274.0	78.3	72.7	82.7	78.0
24	333.3	77.7	71.4	118.5	118.0	370.4	74.3	65.7	81.7	71.9
25	476.2	75.0	80.0	120.6	100.1	512.8	72.7	64.6	84.0	75.6



(a) Compressive Load vs Axial Displacement

(b) Compressive Load vs Out-of-Plane Displacement

Figure 6: Comparison of Typical Load-Deflection Curves

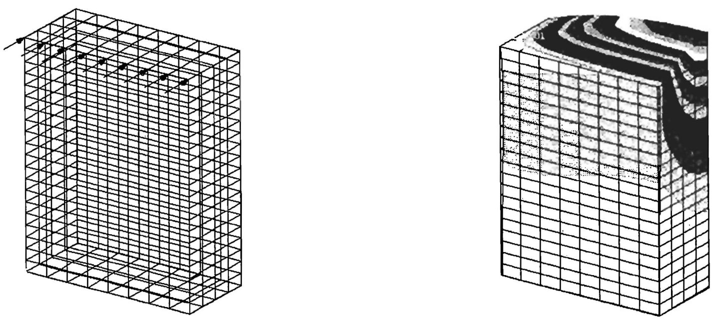


(a) G550 (b) G250

Figure 7: Comparison of FEA and Experimental Ultimate Stress Results

Half Wave Buckle Length Model

The sandwich panel used in the experiments does not represent exactly the panels used in various structural systems. For the simplicity of the experiments, foam width was made the same as the steel face width. In the test rig, only the steel faces were restrained along the four sides leaving the foam unrestrained, but the foam in real panels is continuous along the width direction. Hence the half-length finite element model developed to simulate the experimental panels cannot be used for reviewing and developing the design rules for local buckling of sandwich panels. However, the validation of half-length model by comparing its results with the experimental results provided the confidence in using realistic FEA model for developing design rules. The half-wave buckle model agrees matches with the theoretical model used to develop the buckling stress formula based on elastic half space method (Equations 1 to 5). Hence a single half-wave buckle was modelled with appropriate boundary conditions including that of symmetry.



(a) Model (b) Buckle shape

Figure 8: Half-Wave Buckle Length Model of Steel Plate with Foam Core

Appropriate boundary conditions were applied to the entire surface (i.e., to the steel faces and foam core) along all four sides. The length of the half-wave buckle model, $a/2$, was found by varying $a/2$ until the minimum buckling stress was obtained. The width of the model was $b/2$ (half the plate width), length $a/2$, and thickness sum of the foam and steel thicknesses ($t_c + t_f$). The model geometry and the mesh used in the analysis are shown in Figure 8a. As in the case of half-length experimental model, the half-wave buckle length model was also analysed first using elastic buckling analysis, followed by non-linear analysis. The first buckling mode from the elastic buckling analysis was used to input geometric imperfection for the non-linear analysis with a magnitude of $0.1t$. Figure 8b shows the buckled shape of the half-wave buckle length model.

Table 3: Comparison of FEA Results based on Half-Wave Buckle Length Model with Theoretical Results for G550 Steel Plates

Test series	b/t ratio	a/b ratio		Buckling stress (MPa)		Buckling stress ratio	Ultimate stress (MPa)
		Theory	FEA	Theory	FEA	FEA/Theory	FEA
1	52.6	0.883552	0.88	350.3	350.9	1.00	377.5
2	62.5	0.833907	0.84	275.8	274.8	1.00	334.0
3	83.3	0.727277	0.72	196.7	194.3	0.99	270.3
4	119.0	0.576463	0.60	146.7	143.3	0.98	228.6
5	84.2	0.717689	0.73	189.5	185.0	0.98	250.3
6	100.0	0.646209	0.65	163.3	158.8	0.97	223.0
7	133.3	0.526465	0.55	135.6	130.8	0.96	189.6
8	190.5	0.393099	0.40	117.6	113.4	0.96	188.7
9	105.3	0.621449	0.64	155.1	149.9	0.97	210.5
10	125.0	0.550283	0.56	139.1	134.0	0.96	187.3
11	166.7	0.438809	0.46	122.2	117.2	0.96	162.3
12	238.1	0.322085	0.34	111.0	106.7	0.96	161.0
13	126.3	0.54359	0.57	137.0	131.6	0.96	182.9
14	150.0	0.476173	0.48	126.5	121.1	0.96	162.9
15	200.0	0.374781	0.38	115.0	110.0	0.96	155.4
16	157.9	0.454197	0.47	122.6	117.1	0.95	154.4
17	187.5	0.394146	0.41	116.2	110.9	0.95	143.8
18	250.0	0.306453	0.32	109.2	104.3	0.96	141.9
19	357.1	0.220433	0.23	104.6	100.6	0.96	122.7
20	300.0	0.258671	0.27	106.1	101.3	0.95	124.7
21	428.6	0.184972	0.19	103.2	99.2	0.96	113.0
22	210.5	0.353945	0.37	111.6	105.9	0.95	133.4
23	250.0	0.304439	0.32	108.3	103.1	0.95	132.6
24	333.3	0.234151	0.24	104.8	100.0	0.95	117.1
25	476.2	0.16699	0.17	102.5	98.6	0.96	108.9

The half-wave buckle length a and critical buckling load were obtained from elastic buckling analysis whereas the ultimate failure load was obtained from non-linear analysis. Half-wave buckling length a and critical buckling load were compared with the theoretical results obtained from Equations 1 to 5. Table 3 presents the comparison of these buckling results from FEA and theory along with the ultimate load obtained from the FEA for G550 steel plates. As seen from the table, both half-wave buckle length a and critical buckling loads from FEA agreed reasonably well with the theoretical results. The mean and COV of the ratio of buckling loads between FEA and theory was found to be 0.97 and 0.01, respectively. Hence these agreements confirm that the half-wave buckle length model can be successfully used to model the local buckling behaviour, review the existing design rules, understand the inadequacy of current effective width approach for slender plates, and develop new improved design formulae.

DEVELOPMENT OF NEW DESIGN RULES

The buckling coefficient K was determined for the steel plate elements supported by foam core by using Equations 8, 10, and 11 proposed by Davies & Hakmi (1991), Mahendran & Jeevaharan (1999), and ECCS (2000), respectively. Theoretical values of K were evaluated using Equation 2. The K values obtained from various formulae were then utilised to determine the slenderness parameter λ in Equation 12. The purpose of evaluating K and λ is to determine the effective width of the steel plate elements required in the design. Hence the available formulae should be able to predict accurate values of effective width. By substituting K and λ in Equation 12, the effective width of the steel plate element was determined. This effective width approach is the current design rule for cold-formed steel members and has been extended to the foam supported steel plate elements (ECCS, 200). On the other hand, effective width of the foam supported steel plate element was also determined from the ultimate strength obtained from the finite element analysis using b_{eff}/b = ultimate stress/yield stress. Effective width evaluated based on different buckling formulae together with the FEA results are plotted against the b/t ratios in Figure 9.

As seen in Figure 9, the effective width values evaluated from various design equations agreed reasonably well with the FEA results for low b/t ratios (< 100). But for higher b/t ratios all the formulae predict very high effective width values compared with FEA results. So for slender plates, none of the design methods using different buckling formulae could estimate reasonable values of effective width b_{eff}. This comparison clearly shows and confirms that the current design formulae are not applicable for slender plates.

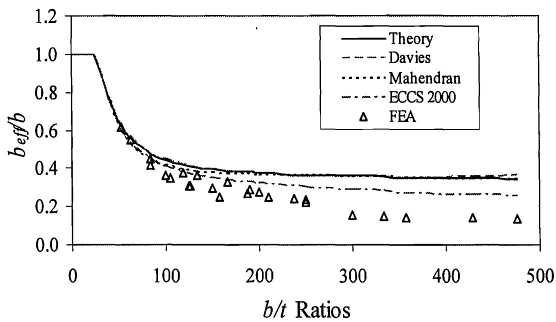


Figure 9: Effective Width of the Steel Plate Elements Stiffened by Foam for G550 Steel

From these results, it can be concluded that improved design formulae have to be developed for slender plates based on the finite element analysis results. Hence, the FEA results for all the specimens were evaluated and some more FEA were undertaken to include b/t ratios from 30 to 600. This produced a larger database covering a wider range of b/t ratios for sandwich panels subject to local buckling effects. Based on these FEA results, an improved design equation has been proposed next for the design of sandwich panels.

Effective width of foam supported plate element can be given as:

$$b_{eff} = Ct\sqrt{\frac{KE_f}{f_y}} \tag{13}$$

The following equation has been developed for the term C based on FEA results:

$$C = 0.2831\left[1 + 11.12\left(\frac{t}{b}\right)\sqrt{\frac{E_f}{f_y}} - 23.44\left(\frac{t}{b}\right)^2\left(\frac{E_f}{f_y}\right) + 15.13\left(\frac{t}{b}\right)^3\left(\frac{E_f}{f_y}\right)^{3/2}\right] \tag{14}$$

By substituting the value of C in Equation 13, a modified formula for computing the effective width b_{eff} for foam supported plate elements can be obtained as below:

$$\frac{b_{eff}}{b} = \frac{0.298}{\lambda}\left[1 + \frac{11.70}{\beta} - \frac{25.97}{\beta^2} + \frac{17.65}{\beta^3}\right] \tag{15}$$

where

$$\lambda = 1.052\left[\frac{b}{t}\right]\sqrt{\frac{f_y}{E_f K}}$$

$$\beta = 1.052\left[\frac{b}{t}\right]\sqrt{\frac{f_y}{E_f}}$$

Figure 10 presents the comparison of effective widths obtained from the FEA results and predicted from Equation 15. This improved equation provides a very good agreement with the FEA results. This comparison also confirms that Equation 15 can be used for compact plates with very low b/t ratios to the slender plates with very high b/t ratios (up to 600). However, further research is currently under way to validate and improve this approach.

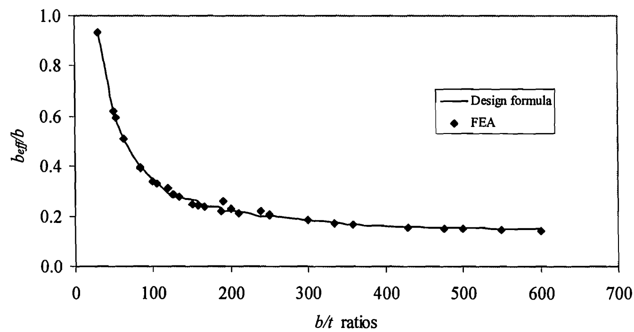


Figure 10: Effective Width Evaluated Based on Modified Equation

CONCLUSIONS

An extensive series of experiments and finite element analyses was conducted to investigate the local buckling behaviour of foam supported steel plate elements. Appropriate finite element models were developed for the simulation of experimental panels and more realistic panels used in various building structures. Numerical results were than used to review the existing design rules. They reveal the inadequacy of using the conventional effective width approach. It is concluded that for low *b/t* ratios (<100) current design rules can be applied, but for slender plates these rules can not be extended in their present form. Based on the finite element analyses results, an improved design equation has been proposed for wider range of *b/t* ratios (<600). Further research is under way to prove its adequacy.

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